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Design principles for pile supported piers.

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DESIGN PRINCIPLES
FOR
PILE SUPPORTED PIERS

by

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1. INTRODUCTION

A pier is a structure which extends from the land outward into the water at some angle, usually close to ninety degrees, with the shoreline. A marginal wharf is a structure which is built parallel with the shoreline. The present-day cargo handling procedure is more efficient at a marginal wharf due to the increased area for truck maneuvering in the upland; however, many ports do not have sufficient waterfront space to use marginal wharves to the exclusion of piers. A type of pier which combines the features of a pier and a marginal wharf is finding increasing favor among port operators. It has a nearly square plan with a "U" shaped transit shed extending along the sides and across the outboard end and there is an open truck court in the center. This pier provides three berths, one on each side and one at the outboard end. While not quite as convenient for cargo operations, the U-shaped pier requires less waterfront space per berth than a marginal wharf and is actually more saving of space than a finger pier, except in the case of very long, narrow piers. Although for very wide piers a solid fill structure might be more economical, established bulkhead lines will preclude solid structures and it is safe to say that most of the future construction in established ports will be pile supported piers.

No attempt is made to give design and construction details nor are typical designs presented in this thesis. It is felt that there is no such thing as a "typical" design to fit all conditions and the references listed herein provide the reader with adequate examples of piers which have been constructed in the past. Nothing would be gained by duplicating such drawings in a work of this nature. Design and construction details are of such variety that a work of this length could not cover all the possibilities which might be encountered in practice. Details are best learned through practical experience.

As part of the work on this thesis, an attempt was made to determine the effect on the cost of construction of such things as pile spacing, pile types, type of deck construction (i.e. beam and slab vs. flat slab) and size of pier. It was soon realized that there are so many variables involved that such a study would only be applicable to one particular set of site conditions in one particular port and no generalizations would be possible. This is discussed further in Chapter 7.

So much emphasis is placed on the necessity for adequate subsurface investigation that it hardly seems necessary to repeat it here. There is no minimum or maximum number of borings required. More variable subsurface conditions will require a greater number of borings. A study of past construction in the area is always necessary.

There is one point which is deemed worthy of special emphasis. The one thought which should be foremost in the designer's mind is "simplicity." Simple construction procedures will result in lowest possible construction cost. Intricate systems of bracing and elaborate fendering systems, while theoretically more desirable, will only cause increased maintenance problems for the owner. Fendering systems involving hydraulic shock absorbers and/or joints which require lubrication or other periodic maintenance other than the replacement of worn or damaged members should be avoided.

2. PILE DRIVING, PILE FORMULAS AND PILE LOAD TESTS

2.1 Static Formulas

There are basically three ways of estimating the bearing capacity of piling, i.e. static formula, dynamic formula and static load test.

Static formulas of great variety have been used in the past and still new ones are being proposed. Some are based simply on adding the estimated tip resistance and skin resistance. The point resistance is calculated using Terzaghi's formula for deep foundations[50, p.172] and the skin resistance is calculated by assuming either a constant friction value for the entire depth of penetration or friction which increases linearly with depth. Chellis[9, p.42] lists some constant friction values which are encountered in practice. As an example of a recent proposal, A. J. da Costa Nunes, in a paper presented at the Third International Conference on Soil Mechanics and Foundation Engineering,[16] proposed a formula which determined the skin resistance by using the angle of friction between the pile and soil and used the coefficient of lateral earth pressure at rest to determine the lateral pressure against the sides of the pile. He calculated the point resistance by Terzaghi's criteria for a deep footing. Formulas which consider skin friction and point resistance separately do not account for the fact that the shear stresses along the

sides of the pile will increase the vertical stress on the plane of the pile tip, thereby increasing the resistance of the soil to failure in that plane. Also the skin friction may not vary linearly with depth and it will almost certainly not be uniform with depth. These formulas also ignore the effects of adhesion between the pile and the soil which will occur in clay soils.

Nishida recently proposed a formula based on the theory of elasticity[38] and Kezdi developed a semi-empirical formula based on the relation of skin friction to strain for cases of piles embedded in sand.[25] Formulas of this type fundamentally more accurately represent the actual stress conditions than the type which consider point resistance and skin resistance separately. However, soils are not perfectly elastic and at the present state of our knowledge of soil mechanics it is not possible to determine the soil constants with sufficient accuracy to make the use of static formulas reliable for anything more than a rough estimate. This is especially applicable in attempting to evaluate the coefficient of lateral earth pressure. A displacement pile will increase the lateral stress in the soil by an indeterminate amount so that it will be something between the neutral and the passive earth pressure. One should keep in mind, however, that if the soil constants can be determined from actual pile

tests on previous work in the same type of soil, a static formula can be of real value.

2.2 Dynamic Formulas

Dynamic formulas are more numerous and are more widely used than static formulas. The fact that they are more widely used does not mean that they are any more reliable. The basic assumption of all dynamic formulas is that the energy of the hammer is related to the ultimate resistance of the pile multiplied by the average set of the pile for the last few blows of the hammer. The simpler formulas attempt to account for energy losses by large factors of safety and certain constant coefficients. Others introduce factors including the relative weights of pile and hammer. The more elaborate formulas attempt to evaluate losses on the basis of Newtonian impact and elastic strain energy of driving cap, pile and soil. This latter group of formulas are of the Hiley type and are based on the assumptions that Newton's law of impact applies to pile driving, that the pile is under compression throughout its length and that the stress is promulgated instantaneously throughout the pile. None of these assumptions are strictly correct but they are nearly true for relatively short, light piles. Attempts have also been made to develop strictly empirical formulas based on a study of pile driving records.[18] The number of formulas alone is evidence that

there have been wide differences of opinion as to the correctness of such formulas.

Along with the fact that the dynamic formulas are not theoretically correct, another serious objection to their applicability is that they do not take into consideration the nature of the soil into which the pile is driven. In a sensitive clay, the effect of remolding or of lubrication by free water will result in a low resistance to penetration while the pile is being driven. After driving is completed and time is allowed for the clay to reconsolidate, the bearing capacity will be appreciably increased due to the gain in shear strength of the clay and to the development of bond between the clay and the pile surface. Redriving of piles in clay and applying the formula to the set at the beginning of the redriving will give some indication of the increase in bearing capacity.

On the other hand, in some compact silts, fine sands and some non-sensitive clays, the permeability is such that the free water cannot escape as the soil is compressed by the pile tip and the driving resistance may be greatly increased by the pore water pressure built up. After a certain rest period, the pressure will be relieved and the pile can be redriven with a greatly increased set. Calculating bearing capacity by dynamic formula under such conditions may result in errors on the unsafe side.

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Redriving of piles (after a period of rest) in this type of soil is the only way to achieve full bearing capacity.

A loose sand or silt may exhibit thixotropic properties and under the vibration of the hammer a very small resistance will be experienced, but when driving is stopped, the soil will resume its stable structure and exhibit greatly increased bearing capacity. Redriving piles in such a soil would not indicate any increase in resistance since the vibration would immediately cause the unstable state. A static load test would be the only reliable means to ascertain the safe bearing capacity.

Negative friction of a consolidating layer must also be evaluated. This can be caused by consolidation of newly placed fill, from reconsolidation of the clay disturbed by the pile driving, or by consolidation of an existing layer by an increase in load on the surface. The piles in the majority of piers would be affected only by reconsolidation of disturbed clay, but those piles near the shoreward end might be affected by any of the above causes.

By way of recapitulation, the dynamic pile driving formulas in general use today are not theoretically correct and cannot be expected to give accurate results under all conditions. The more complete formulas of the Hiley type give fairly reliable results in conditions where the principal support comes from sandy soil and where the pile length is not excessive, say less than 70 feet. The

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formula should not be applied to extremely heavy piles or extremely long piles. The Engineering News formula, while more generally used in this country, gives more erratic results than formulas of the Hiley type. The Engineering News formula can be expected to give fairly reliable results in light piling driven into sandy soils, when the final set is not smaller than 1/4 inch per blow.[10] No dynamic formula can be reliable in a soil where time-dependent factors influence the bearing capacity, i.e. soils that either relax or set up after pile driving is complete.

2.3 Static Load Tests

The most reliable way to determine the safe bearing capacity of a single pile is by static load test. Basically two methods can be employed. The load can be applied by weights such as iron ingots or concrete blocks. The weights could be placed directly on a platform attached to the pile, but normally the weights are supported on a platform and the load is applied to the pile by jacking against the platform. The other method is to use two or more reaction piles connected by a beam, the load being applied to the pile by jacking against the beam. This method involves a rather complicated stress interaction in the soil, the effect of which cannot be evaluated. Normally the reaction piles are kept five to eight feet away from the test pile to reduce the effect to a minimum.

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Local codes usually give guides for interpreting pile test results. Frequently there is no clear-cut failure of the pile and the ultimate load must be determined on the basis of a certain maximum net settlement (sometimes 1/4 inch). In evaluating the results of a pile test the soil properties must be taken into consideration. Pile load tests are usually of such short duration that negative friction will not have time to develop. In fact, the soil layers which may ultimately add to the pile load by negative friction will, during the short duration test, actually support part of the load. It is possible to eliminate the effect of compressible strata near the surface by installing a pipe through the compressible layers, cleaning out the pipe and driving the test pile inside. The static load test is by far the most valuable tool for the engineer in determining bearing capacity, but it must be applied with good judgment based on a knowledge of the soil conditions at the site. As to the question of whether the test of a single pile can be extrapolated for the design of the entire structure, only a qualified soils engineer can determine. The results can normally be extrapolated if the supporting stratum is sand; in clays, single pile tests can be extremely misleading. Normally pile spacings for pier construction are great enough that group action is not of concern, however, it must be determined whether underlying strata will be over-loaded, or whether settlement will be

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excessive. Where a stratum of firm sand overlies compressible clay or peat, static load tests can give misleading results. Piles driven into the sand may bear up and perform satisfactorily in the test, but settlement due to the underlying compressible strata may be disastrous to the completed structure. In the preliminary study for a pier design, the engineer should always investigate previous jobs and tests in the viçinity. Information thus obtained may be as valuable as short duration tests and certainly less expensive.

2.4 Wave Theory of Stress Propagation in Pile Driving

The basic theory of stress wave propagation from impact of a weight on a slender rod is not new but the application of the theory to pile driving is so complicated that it has not been of value to the practical engineer until relatively recent years. In 1938 the theory was applied by investigators at the British Building Research Station in connection with a study of stresses in reinforced concrete piling.(20) This investigation revealed much practical information specifically applicable to reinforced concrete piling, but some of the findings would apply to pile driving in general.

The tests at the Building Research Station showed that the stress at the head of the pile is not dependent on conditions in the soil but is dependent only on the

stiffness of the packing and velocity of the hammer, the head stress being about the same whether the driving is easy or hard. On the other hand, stress at the toe and the remainder of the pile is very much affected by the soil conditions. In soft driving, the stress wave reflected from the toe will cause tensile stresses in the mid-body of the pile high enough to exceed the tensile strength of the concrete. If the top of the pile is in a dense surface stratum or crust and the toe in a soft clay, the tensile stresses will be increased. The stress wave is damped by the friction along the pile and in firm material such as sand or gravel the maximum stresses will be approximately the same throughout the length of the pile. With a high toe resistance and little skin friction, the reflected stress wave will set up higher compressive stresses in the lower part of the pile. It is possible for the stress at the toe to be double that at the head. Knowledge of this fact is important in attempting to drive through an obstruction or firm stratum encountered in otherwise soft driving or in driving to refusal in a compact layer with soft upper materials. Under such conditions failure may occur near the tip with no evidence of damage at the head.

In conjunction with the study of stresses, the investigators at the Building Research Station showed that the solution of the basic wave equation could be used to explain the phenomena which they had observed. If, however,

all the variables are included, the solution of the wave equation becomes so complicated that solution by ordinary means is virtually impossible. In recent years the electronic digital computer has enabled engineers to solve the wave equation by the method of difference equations, taking into consideration all the variables that are known or can be assumed.(43) Although the variables such as coefficient of restitution of dolly and packing, elastic-plastic properties of the soil, pile material, shape and length, can all be taken into consideration in the calculation, it is unlikely that a single formula will be developed which would be universally applicable. It will, however, be possible to present the calculations in useful form by graphs. The Raymond Concrete Pile Company in conjunction with IBM is doing considerable research in this regard.(44) The method of calculation used is explained in reference 42 and is applicable to either electronic computer or ordinary desk calculator.

If all the variables, including the properties of the soil, could be accurately determined, the solution of the wave equation would give exact bearing capacity of the pile. Unfortunately these determinations are not now possible and it may be some time in the future before our techniques are improved enough to make accurate determinations. This, however, does not detract from the fact that at least we now can approach the pile driving problem with

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the correct theory, something which cannot be said for the present dynamic formulas.

As in the case of any dynamic formula, accurate results cannot be expected in soils where the bearing capacity varies with time. At best a dynamic analysis, even one theoretically correct, is useful only as an additional source of information to be combined with subsurface soil data and pile load tests to aid the engineer in designing an adequate foundation. Since it is obviously impractical to test every pile in a foundation, the dynamic analysis is a tool which, along with known depths to bearing stratum, can be used to estimate when the piles have been driven to the required bearing capacity by fitting the dynamic analysis to the observed results of the load test.

2.5 Common Pile Loads

The previous discussion has considered only the bearing capacity of the pile as regards the portion embedded in the soil. Of equal concern in the design of piling are handling stresses and the unsupported length of pile above the dredge line. These items will be discussed in subsequent sections, however listed below are some generally accepted bearing capacities of piles:

Wood Piles - - - - - - - - -	15-25 tons
Composite Wood & R/C - - -	20-30 tons
Concrete - - - - - - - - - -	30-50 tons
Steel H-beam or Pipe - - -	30-60 tons normally but may be as high as 100 tons
Prestressed Concrete Cylinder Piles 36" diam -	100-200 tons

2.6 Lateral Capacity of Piles

In pier design, accurate determination of the lateral capacity of piles is just as important as the vertical capacity. The pier will be subjected to appreciable horizontal loads from ship impact, wind, current and waves, and realistic values of lateral capacity may permit elimination of batter piles with a considerable saving in cost. Unfortunately, determining the lateral capacity is not as simple as determining vertical capacity.

Because of lack of understanding of lateral capacity, most codes use very conservative values, and designers tend to ignore lateral capacity altogether and rely on batter piles for horizontal loads. The New York City building code allows 1000 pounds per pile unless it is proved by test that the pile will resist 200 per cent of the working load with a lateral deflection of less than 1/2 inch and will resist the working load with less than 3/16 inch deflection at the ground surface. American Civil Engineering Practice states that almost any fully embedded pile will resist a lateral force of 1000 pounds without appreciable movement.[1, p.8-28] In view of test data from papers presented in "Symposium on Lateral Load Tests on Piles,"[47] the value of 1000 pounds is conservative to the extreme. With the limited information available, it is dangerous to generalize but, according to the reported test results, piles driven in firm soil can resist lateral loads

ranging from 4 to 11 tons per pile with deflections not exceeding 1/4 inch at grade.[47] The capacity, of course, varies with the nature of the soil and the size and type of the pile.

It is well established that almost any soil will provide sufficient lateral support to prevent buckling as a long column under vertical load, but this must not be construed to mean that any soil will provide sufficient lateral support for lateral loads. On the contrary, the lateral capacity of a pile for a given deflection is very much dependent on the nature of the soil near the ground surface. Soil layers deeper than about 20 feet have little effect on the lateral capacity of ordinary piles.[47, p.72]

Tests have revealed that increasing the rigidity of a pile will increase its lateral resistance. Also the lateral capacity of a vertical pile is not affected by the amount of vertical load except as regards stress in the pile itself. The position of maximum bending moment is considerably closer to the ground surface than is normally thought and the position is not affected appreciably by such factors as height of thrust above the ground, width of the face of the pile pushed against the soil, stiffness or rigidity of the pile or the depth of embedment. The peak pressure of the soil against the pile is considerably greater than the passive pressure computed from classical earth pressure theories.[48, p.31]

2.7 Theoretical Analysis of Lateral Capacity of Piles

It is possible to conduct lateral load tests on piles but it becomes quite expensive if an attempt is made to provide restraint at the head of the pile to duplicate actual structural conditions. A theoretical analysis has been presented which makes it possible to estimate the capacity of a pile with any degree of fixity at the head from the test results on a free-head pile.[47, p.93]

The theoretical analysis is based on the fundamental differential equations for beams considering the interaction of soil and pile and are as follows:

$$EI \frac{d^4y}{dx^4} = -p = -k\left(\frac{x}{L}\right)^n y \qquad (1)$$

$$EI \frac{d^3y}{dx^3} = -V \qquad (2)$$

$$EI \frac{d^2y}{dx^2} = -M \qquad (3)$$

Where

E = modulus of elasticity of the pile;

I = moment of inertia of the pile cross-section;

y = deflection of the pile at any point along its length;

x = depth of any point below the soil surface;

L = embedded length of pile;

k = modulus of earth reaction at the lower end of the pile;

n = a parameter; any positive, abstract number greater than zero;

p = the net earth pressure at any point along the embedded pile length;

V = shear in the pile at any point along its length;

M = moment in the pile at any point along its length.

Equation (1) is solved for the deflection (y) in terms of the depth (x) to obtain the general solution. Using the known boundary conditions of shear and moment as expressed by equations (2) and (3), a particular solution can be obtained. The solution has been accomplished by the use of calculus only for the cases where n = 0 and n = 1.[47, p.86] Actual values for "n" probably vary between zero and unity and it is possible that for some soils it may exceed unity. Furthermore, soils will rarely be homogeneous and the "n" value will vary among different strata, therefore solution by calculus will not be possible for most cases.

Palmer and Thompson proposed a method of solving equation (1) by the use of difference equations.[39] Their original work was expanded by Gleser[47, p.75] and further elaborated by Palmer and Brown.[48, p.22] By the use of this method it is possible to solve equation (1) for any known soil condition. However, the parameters "k" and "n" cannot be determined by any present soil test, but are obtained by trial and error from the measured pressures,

moments and deflections of a pile load test. Mason and Bishop observed that there are several values of "k" and "n" which will cause either the theoretical deflection or pressure curve to agree with measured values but there is only one set of "k" and "n" which will cause both to agree simultaneously.[48, p.18] Therefore, for practical application of this theory, it is necessary to conduct a pile test and independently measure any two of the items--deflection, pressure or bending moment.

The method of difference equations is straightforward but laborious; fortunately it can be adapted to solution by electronic digital computers.[48, p.11] This method provides the designer with the most rational approach to the design of vertical piles for lateral loads available at this date. It is, however, subject to the limitation that the soil is not perfectly elastic and there will be certain irreversible deflections under repeated loading. Furthermore, as in the case of vertical loads, the total lateral capacity of a pile group cannot be predicted from the capacity of a single pile. In pier construction pile spacing will normally be great enough that interaction between piles will not be of concern.

If batter piles are incorporated in the design, the lateral capacity of the vertical piles should be ignored. Properly connected batter piles will reach the failure load before there would be enough deflection to

mobilize any appreciable portion of the lateral capacity of the vertical piles, therefore it is not possible to add the lateral capacities of the vertical and batter piles. The support of the vertical piles can be considered only as an additional factor of safety of undetermined amount.

With the brief and "sketchy" treatment given above, the writer attempted only to present the basic principle involved and to point out that present design recommendations are over-conservative as regards the lateral capacity of vertical piles. If the reader is interested in pursuing this subject further, the detailed methods of calculation are given in references 47 and 48.

3. TIMBER FOR PIER CONSTRUCTION

3.1 Marine Borers

Timber was used far more in the past than today, before the advent of more durable concrete and steel. However, if it is properly treated, timber can be used for permanent structures. The economic life of timber structures can vary from 15 to 50 years. The primary factors affecting the life of a timber structure are decay, marine borers, storms and fire.

If marine borers are present, they will be the factor of most concern to the designer of waterfront structures. Marine borers fall into two broad categories--the mollusca, including teredo, bankia, martesia, xylophoga and pholadidae, of which the teredo is most prevalent; and crustacea, including limnoria, chelura and sphaeroma, of which limnoria is most important.

The teredo or shipworm most frequently attacks just above the mud line, but attack may occur anywhere below the water surface. Damage is not apparent on the surface under a cursory inspection as the entrance hole may be only 1/100 to 1/30 inch in diameter. The teredo is a greyish slimy worm-like creature varying in length from a few inches to four or five feet and from 1/8 to 1 inch in cross-section. Reproduction usually takes place by free floating eggs ejected by the female which are fertilized by sperm

ejected by the male. In some species, sperm are taken in the siphon of the female and the fertilized eggs are ejected. The larva swims freely and attaches itself to wood where it bores in at right angles for a short distance and then turns parallel with the grain. The tail always remains at the entrance hole and the head, which is equipped with two shells, advances by working the shells back and forth. The tail is equipped with two "pallets" which seal off the burrow during unfavorable conditions. Two siphons for the intake and exhaust of water are located in the tail. The main food of the teredo is the wood itself, but plankton taken in through the siphon is a necessary supplement. The teredo can live for about two weeks if the wood is completely removed from the water.

The limnoria, commonly called gribble, look like minute wood lice. The adult is about 1/8 inch long. For reproduction, the female carries 8 to 12 eggs in a brood pouch. When the young hatch they do not wander very far, but attack the wood close to the point where they hatch. Attack is usually confined to the tidal zone and damage is characterized by "necking" down or hour-glass shape. Attack starts at the wood surface and proceeds inward as the wood is consumed. The surface of the attacked area has a sponge-like appearance and is readily visible on inspection. In some colder climates the zone of attack may be near the mud line rather than in the tidal zone.

Chelura are very similar in appearance and activity to the limnoria. The chelura is always associated with limnoria and apparently does not initiate attack but occurs only in the presence of limnoria.

Sphaeroma are larger than limnoria (about 1/2 inch long) but their attack is similar, being characterized by necking down, generally in the tidal zone but also may be at the mud line. The heaviest sphaeroma attack has occurred in the St. Johns River, Florida, and Lake Pontchartrain, Louisiana. These waters are nearly fresh but the sphaeroma has also been found in sea water of high salinity. The sphaeroma has been found on the Pacific Coast of the United States but damage has not been as serious as in the warmer Atlantic Coast waters.

Salinity is the most important single environmental factor for the occurrence of marine borers. The teredo normally requires more than 10 or 20 parts of salt per 1000, although some species have been found in tropical fresh water; and limnoria requires about 20 to 30 parts of salt per 1000. (Normal sea water averages 30 to 35 parts per 1000.) Temperature is of importance in the breeding season. The teredo must have fairly warm water for the larva to exist, but once it is in the wood activity continues until the temperature is near the freezing point. Teredo activity is highest during the warm season and is

higher in the tropical areas than in temperate. Limnoria is generally less affected by temperature than teredo.

Pollution of certain chemicals, oils, etc. and suspended mud nearly always reduces borer activity. There is a difference of opinion as to the effects of domestic sewage pollution. In some cases it actually appears beneficial, probably by increasing the plankton; but if the pollution is high enough to reduce the oxygen content appreciably, the effect will be detrimental. It is doubtful that any harbor is so badly polluted that the oxygen content will be appreciably reduced. Limnoria appears to be completely unaffected by domestic pollution. Hydrogen sulfide is fatal to marine borers except in extremely small concentration. Sea water is normally slightly alkaline (pH 7.5 to 8.5) and any change either up or down, as by pollution of acid, has a marked effect in reducing borer activity.

3.2 Timbers Resistant to Marine Borers

Some tropical timbers have natural resistance in some waters. However this is by no means a guarantee that they will be immune to borer attack in all waters. ASCE Manuals of Engineering Practice No. 17 lists some of the more resistant woods.[3] Greenheart from Guiana is quite resistant in temperate waters but it has been attacked in tropical waters. Angelique and Manbarklak from Guiana and

Malabayabas from the Philippines are quite resistant to mollusca (teredo) but are attacked to some extent by crustacea (limnoria). Turpentine wood from Australia and Tasmania has shown good resistance, however its sapwood is freely attacked. Other resistant timbers are Totara, Azobe Foengo, Anoura, Sponse Hoedoe, Kajol Lara, Kakala, Kajol Malas and Alcornoque.

3.3 Metal Jackets and Scupper Nailing

Metal jackets and scupper nailing have been used in the past as protection against marine borers. Copper, zinc, and copper-zinc alloy have been found effective but they are subject to damage and theft and are no longer considered economical. Unbroken bark gives temporary protection (a few months). Charring the surface was used by ancients but it is only of temporary value. Various tar and burlap wrappings have been used with limited success, but any such coating has little resistance to abrasion and, once the shell is broken, the pile is freely attacked.

3.4 Concrete Jackets

Concrete jackets have proved quite successful as protection against borer attack. The jackets have been applied to both treated and untreated piles. Precast concrete cylinders and cast in place and gunite jackets have all been used. In some cases the piles were driven butt down and then a concrete cylinder placed over the pile and

driven below the mud line. The annular space is then filled with concrete or sand. Sectional sheet metal forms can be placed around the pile and concrete then placed either by tremie or intrusion-prepacked method. In some cases the bottom of a steel form is closed by steel fingers and canvas which presses against the pile; the form is filled with concrete above the water line and then lowered, successive sections being added to the top as the form is lowered.

Gunite jackets have been effective and have been used to a considerable extent on the West Coast.(54) A wire mesh is placed around the pile and held 3/4 or 1 inch from the pile face by staples and chairs or spacers. The mesh is electrically welded 2 in. by 2 in. no. 12 or 14 wire and is lapped about 6 inches. About six additional turns of no. 9 wire should be placed at the top and bottom of the jacket. Three or four rows of shear notches 4 in. by 4 in. by 1 in. or 4 in. diameter by 1 in. deep are spaced 18 to 24 inches apart along the pile to prevent the pile from sliding inside the shell. The shell is $1\frac{1}{2}$ to 2 inches thick and extends from about 2 feet above high water to 5 feet below the mud line. It was found advisable to wet the pile and keep the mortar as dry as possible. Gunite jacketed piles are better able to resist hard driving than ordinary wood piles and in one case performed better than precast concrete piles.(36) These piles have served well and were economical in the past when the

durability of ordinary reinforced concrete left much to be desired. However, with the increasing use of prestressed concrete, it is questionable whether they will be used extensively in the future.

3.5 Creosote Treatment

Preservative treatment with coal tar creosote or creosote-coal tar solutions is the most common and is the best available treatment for wood piles at present. Treatment should be by the full cell, pressure process in accordance with specifications AWPA C-3 or Federal Specification TT-W-571c. A. P. Richards of Clapp Laboratories has stated that where limnoria are involved, 70/30 creosote/coal tar solutions in accordance with ASTM Designation D 391-53 or AWPA P-2 give better results than distillate or low residue distillate creosote.(40) Richards recommends that minimum specific gravities be increased above those recommended in ASTM Designation D 390-53 and D 391-53, AWPA P-1 and P-2, or Federal Specification TT-W-556C. Where 1.025 is now acceptable, 1.030 should be specified; and where 1.085 is now acceptable, 1.100 should be specified.(40)

For piling, minimum recommended retentions are 16 lb. per cu. ft. for Douglas Fir and 20 lb. per cu. ft. for Southern Yellow Pine. Timber above mean high water should have retentions as recommended by AWPA standard C 18-59. In all cases retention should be determined by

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the "assay" method developed by the Forest Products Laboratory at Madison, Wisconsin. Richards also recommends that if more than ten per cent of the piling in any given charge have inadequate retention, the entire charge should be rejected.(40)

Sawn timber, since the easily treated sapwood is removed, is more difficult to treat and generally more subject to marine borer attack. It is therefore desirable to keep all bracing above the MHW level. Lateral stability should be provided by batter piles, eliminating bracing altogether unless it is necessary to use braces to reduce the unsupported length of piles. Cutting and boring, as far as possible, should be done before treatment. All job cut surfaces should be given at least two coats of hot creosote and all bolt holes should be treated with the Greenlee bolt hole treater, which is a patented device specifically designed for this purpose.(24, p.212) All sawn timber should be incised. The specification should prohibit the use of cant hooks or handling devices which would puncture the creosote shell and should require that all holes, chips, etc. be plugged with creosoted wood.(28)

Piles should be specified in accordance with ASTM Designation D 25-58, and dimension timber should be specified in accordance with the specifications of the association under whose rules it is graded.

3.6 Fire Resistance

To increase the fire resistance of timber piers, underdeck fire bulkheads should be spaced at 150 to 200 ft. intervals, extending from the underside of the deck to one foot below mean low water. On long piers, complete fire-walls, extending up through the superstructure, should be spaced at 450 ft. intervals. Underdeck bulkheads should be of reinforced concrete or tropical hardwood which is resistant to marine borers. Firewalls should be of reinforced concrete. Certain chemicals, when injected by the pressure process, impart marked fire resistance to timber. Some of these chemicals are chromated zinc chloride, ammonium sulfate, boric acid, diammonium phosphate, sodium tetraborate, and sodium bichromate. These chemicals are available in various commercial preparations and retentions vary from 1.5 to 6 pounds per cubic foot depending on the formulation. All are soluble in water and will leach out if exposed to the weather, consequently are not dependable for the parts of marine structures exposed to weather or salt water splash.

Piers with timber decks should always be provided with underdeck sprinkler systems. Dry pipe systems with rate-of-rise control are most satisfactory. The substructure should be so designed that fire-fighters have access to the underside of the deck by using rafts or boats. Also access holes should be provided through the deck at about

25 ft. intervals for the access of fire-fighting equipment. The Port of New York Authority is fireproofing the wooden piles in its new piers by placing a concrete collar from the mid-tide elevation to the underside of the deck.

3.7 Details

Design details are of such variety and are so much subject to local practice that the subject cannot be discussed within the scope of this thesis. A few comments are considered appropriate, nevertheless. Simplicity of details should be of primary concern in order that construction will be simple and replacement of deteriorated or damaged members can be made with a minimum of difficulty. Reference 49 is an excellent guide for the design of timber connector joints. When timber connectors are used, heavy spiked grids and malleable iron rings are preferable, since thin metal rings and pressed steel plates are subject to rapid deterioration in the marine environment. All bolts and hardware should be galvanized.

Hardwoods are more resistant to mechanical wear and are preferable for deck surfaces. Treated Black Gum is one of the most resistant of woods in America. It decays rapidly if untreated but takes treatment well. It has a tendency to twist badly, especially in thicknesses less than three inches and must be well fastened.[30]

Solid laminated timber decks with asphaltic-concrete or reinforced concrete wearing surfaces have been found successful. By the use of suitable shear connectors, a timber-reinforced concrete deck may be designed for composite action.(30) This type of deck construction has the disadvantage that repair or replacement of individual members is virtually impossible without tearing out a large area of the deck.

Rider caps drift-pinned into the top of the piles are preferable to girder caps which are dapped into and through-bolted to the top of the pile. Uplift of the rider cap can be prevented by steel straps over the cap and bolted to the pile. Bolts are normally the weakest part of any connection, especially if shear plates or spiked grids are not used. In some cases, as in fastening batter piles to vertical piles, U-bolts may make a more satisfactory connection. Spiked grids should be used to transmit the shear. Dimension timber should be designed in accordance with National Design Specification for Stress-Grade Lumber and its Fastenings.(37)

4. CONCRETE FOR PIER CONSTRUCTION

4.1 Deterioration of Concrete

Good concrete is almost an ideal material for pier construction. Piles can be manufactured in an unlimited variety of shapes and sizes and concrete is adaptable to either cast-in-place or precast construction. There are many records of excellent durability of concrete in sea water, but there are unfortunately also many instances of failure. It is the statement of the obvious to say that failures are attributed to poor quality concrete. While it is true that the quality of cement has improved greatly since it was first used in marine work, past failures are in general the result of faulty techniques in making the concrete itself, not from the inadequacy of the cement. The two greatest sources of deterioration of concrete are freezing temperatures and corrosion of the reinforcing steel. Freezing and thawing results in disintegration of the surface, spalling of the concrete and eventual exposure of the reinforcing steel. When the steel corrodes, the corrosion products expand, causing the concrete shell to burst, thus exposing the steel to further attack. Deterioration of concrete piling is normally greatest from about half tide up to a few feet above high water.

One of the most extensive testing programs on the durability of concrete exposed to a marine environment has

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been conducted by the Waterways Experiment Station of the Corps of Engineers. Identical samples were exposed at Treat Island, Cobscook Bay, Maine, and at Salt Run, St. Augustine, Florida.[12] The samples at Treat Island are exposed at the half-tide elevation and during the winter months are subjected to freezing and thawing during each tidal cycle. The water temperature is quite uniform throughout the year, ranging only from 34° to 40° F. During the winter, the samples are thawed and raised to a temperature of about 37° F when submerged and are frozen in air to temperatures ranging from 28° F to -10° F. The deterioration at Treat Island is almost exclusively the result of freezing and thawing; chemical action is apparently insignificant due to the low water temperature.

The samples at St. Augustine, Florida are also exposed at the mid-tide elevation. Here the principle agent of attack is chemical action of the warm sea water.

The significant conclusions from these tests were as follows:

1. Use air entrained concrete in structures exposed to freezing and thawing. Some samples of otherwise good quality with no air-entrainment failed in one winter.
2. In warm sea water Type II cement with less than 8 per cent tricalcium aluminate should be used.

Although the tests indicated that the influence of sulfates in the sea water was insignificant in cold

been conducted by the [illegible] [illegible] [illegible] of the Corps of Engineers. [illegible] [illegible] [illegible] [illegible] Trust School, [illegible], [illegible] and [illegible] [illegible] [illegible] [illegible]. The [illegible] [illegible] [illegible] [illegible] [illegible] of the [illegible] [illegible] and [illegible]. [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible]. [illegible].

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1. [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible].

2. [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible].

[illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] of [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible]

waters, it would be advisable to specify Type II cement for all waterfront work.

A paper written by Wakeman et al.,[52] along with the discussions, gives much practical information from a large number of engineers on the durability of concrete, and gives actual performance records which supplement the information of the tests conducted by the Waterways Experiment Station. The authors recommend a minimum of 3 inches cover over the reinforcing steel. This figure was criticized by some engineers as being over-conservative. W. P. Kinneman[52, p.1326] recommended 2 to $2\frac{1}{2}$ inches for conventional reinforced concrete and $1\frac{1}{4}$ to 2 inches for prestressed concrete. Wentworth-Sheilds and Gray also recommend a minimum of 2 inches cover for sea water exposure.[53, p.53] The Portland Cement Association recommends 3 inches.[11] It would appear that with excellent field control and inspection along with skilled workmanship, 2 or $2\frac{1}{2}$ inches of cover might be adequate, but in view of the possibility of workmanship and inspection less than the optimum, the more conservative 3 inches would be desirable. Evidence was also presented[52, p.1312] which indicated that cements with high tricalcium aluminate contents (up to 17%) have been used in San Francisco harbor with excellent results. It must be realized, however, that the water in San Francisco harbor would not be considered warm sea water.

Reactions have occurred (notably on the West Coast) between the alkalis of the cement and certain siliceous aggregates. The compound formed has greater molecular volume than its constituents, thus the concrete swells and disintegrates. Such reactive aggregates should be avoided, but if they must be used, low-alkali cement must be used and certain pozzolanic admixtures have been found beneficial. Since the exact influence of pozzolanic admixtures cannot be accurately predicted, local experience in using reactive aggregates provides the best guide. ASTM Designations C 227-58T and C 299-57T are recommended standard tests for determining potential alkali-aggregate reactivity.

All authorities agree that where the structure is subjected to freezing temperatures air-entrainment is a necessity. Three to six per cent appears to be the normal range of air content. Not only is the entrained air beneficial in increasing resistance to freezing and thawing, but it will produce a more workable mix and permit the use of a lower water-cement ratio. Wakeman et al. recommend[52] a maximum water content of 6 gallons per sack of cement. Fluss and Gorman[52, p.1320] recommended $4\frac{1}{2}$ gallons per sack for piles and pile jackets and 5 gallons per sack for the remainder of the substructure including the deck. These latter figures appear quite low and such a mix might have such a low slump as to present difficulties in placing.

4.2 Design of Concrete Mixes

The Portland Cement Association provides an excellent guide for the design of concrete mixes (see ref. 17). Particular attention should be paid to the gradation of the fine aggregate. Since it is imperative to have an impermeable concrete, the specification should require a minimum of 2 to 5 per cent of minus 100 mesh material. The gradation curve should have a flat slope indicating a well-graded aggregate. Fine aggregate must be free from organic impurities or plastic fines. ASTM Designation C40-56T should be used as a test for organic impurities in sand. It is, of course, necessary to make the final selection of a mix on the basis of trial batches. There is no way to design for workability from charts or tables.

4.3 Curing of Concrete

Proper curing of concrete is no less important than the design of the mix. Proper curing means the maintenance of controlled conditions for some definite period of time following placing and finishing to assure hydration of the cement and hardening of the concrete.[2] Curing involves the preservation of adequate water content, maintenance at some relatively uniform temperature above freezing, freedom from damaging mechanical disturbance and allowing time for the concrete to gain sufficient strength for the safe use of the structure.

Preservation of adequate water content is accomplished by ponding with water, continuous spray with water, or by application of impervious coatings, membranes or coverings. Forms alone are inadequate to prevent loss of moisture. Normally preservation of moisture is of greater concern than temperature control, since concrete will harden over a wide range of temperatures from just above the freezing point to just below the boiling temperature of water, but it will not harden at any temperature in the absence of water. The rate of hardening increases rapidly with an increase in temperature; however there is evidence that at temperatures above 165° F the ultimate strength is adversely affected.(41)

For cast-in-place concrete, curing at temperatures considerably above the average annual temperature will result in excessive shrinkage. For precast units such shrinkage is normally of little concern since the unit will be at atmospheric temperature, and shrinkage will have already occurred when it is actually placed in the structure.

For protection against low temperatures, the water and/or aggregates can be heated and the concrete can be protected after placing by heated or unheated shelters or insulated coverings. Acceleration of the curing period can be accomplished by the addition of not more than two per cent (2# per sack of cement) calcium chloride. The calcium chloride should not be added in flake form but

should be mixed with water at the rate of one pound per quart and added as part of the mixing water. Calcium chloride should not be used where sulfate resistance is important and it should not be used in prestressed concrete. The effect of two per cent calcium chloride in increasing early strength is proportionately greater at lower curing temperatures than at higher temperatures. There appears to be no advantage in using it if the atmospheric temperature is above 40° F.

Protection against high temperature is considerably more complex than protection against low temperature. Excessive temperature rise may be avoided by using low heat cement, reducing the cement content or precooling the water and/or aggregate. Ideally the curing temperature should be somewhat less than the average annual temperature to which the structure will be subjected during its life. It must be recognized, however, that in most situations it will be impractical to attempt to maintain temperatures in the concrete less than the existing atmospheric temperature.

While not specifically a problem of curing, it has been observed that concrete placed during hot weather is of lower strength than the same mix placed in cooler temperatures. This has sometimes been erroneously attributed to "hot cement" since the warm months are periods of high demand and the cement may be used soon after it is ground with little time in storage to cool. The slump of

a concrete mix decreases when the temperature rises. The actual explanation for lower strength concrete in hot weather is that very often more water is added to maintain a constant slump without adding cement. The water-cement ratio is increased and the strength falls off. "Hot cement" has no effect on the strength other than its effect in raising the temperature of the entire batch.

Laboratory tests have shown[27] that increasing the curing temperature above some optimum (varies with type of cement from 50 to 55° F) will result in higher strength at early stages (up to 7 days) but lower ultimate strength at three months and one year. At normal temperatures this has little practical significance but at temperatures above 165° F the strength reduction may be appreciable.

ACI Committee 612 Report "Curing Concrete"[2] proposed optimum curing practice for various types of concrete construction. The procedure applicable to normal pier construction would be as follows:

> INITIAL CURING (for first 24 hours or a minimum of overnight).
>
> Cover with two thicknesses of woven fiber mat, quilted fiber mat or other absorptive material thoroughly saturated when applied and kept continuously wet by spraying until removed.
>
> FINAL CURING (about 72 hours if the temperature is above 40°F. When the temperature is below 40°F the concrete should be maintained at a temperature of 50° to 70°F for the same period.)
>
> 1. Continuation of same procedure as for initial cure.

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[illegible] as follows:

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2. Two inches of moist earth or sand blanket kept saturated by spraying.

3. Three inches of moist, cured hay, grass or straw kept wet by spraying.

4. Impervious light-colored paper or plastic covering laid directly on the concrete surface.

5. Sprayed-on liquid coating. For surfaces exposed to the sun in hot weather a resin type light-colored compound should be used to reduce the amount of heat absorbed from the sun. For surfaces not exposed to the sun, asphaltic compounds are suitable.

If for practical reasons the fabric is not available or the water spraying would interfere with adjacent construction operations, the best alternative would be to use impervious light-colored paper or plastic or an impervious sprayed-on coating applied immediately following the finishing operation. Sprayed-on coating should be applied at the rate recommended by the manufacturer and must be applied uniformly to insure complete sealing of the surface.

At the end of the curing period the concrete should be allowed to gradually approach the exposure temperature. The drop in temperature should be less than $50^{\circ}F$ during the first 24 hours for thin sections, and for heavy sections (greater than two feet in least dimension) should be less than 30° in the first 24 hours. The maximum rate of change should be no greater than 5° per hour for thin sections and 3° per hour for heavy sections.[2]

Piling should be air-cured for as long a period as possible before driving. The carbon dioxide of the air combines with the calcium hydroxide in the concrete and

forms a shell of calcium carbonate which is very resistant to sea water. Practical considerations will limit this air-curing period but Stroyer states that from six to eight weeks are required for the formation of this skin.(46, p.6)

4.4 Handling of Concrete Piles

Concrete piles must be handled with due care to prevent severe cracking. Usually handling stresses will govern the design of piles rather than the structural stresses; however in deep water where the unsupported length is great the opposite may be true. Long piles should be designed for pickup at as many points as necessary to prevent over-stressing the pile. Reference 11, p. 30, gives the maximum bending moments for 1, 2, 3, 4 and 5 point pickup with recommended locations of the pickup points. To assure that field personnel handle the pile as it was designed, it is advisable to cast lifting eyes into the pile at the specified pickup points.

Steel stresses should be low. The design should be based on 100 per cent impact for handling stresses and if an abnormal amount of handling is involved, it may be desirable to reduce the design steel stress from 20,000 psi to 16,000 psi, still using 100 per cent impact.

Handling stresses, and handling weight, may be reduced by using a hollow section. From the cost standpoint there may be no advantage in using a hollow pile,

since the void may cost as much as the concrete it displaces. However, if handling stresses can be reduced and/or lighter handling equipment can be used, the hollow pile will be justified. Such determination can be made only on the basis of applicable costs and equipment availability.

4.5 Driving Stresses

Since driving stresses will normally be greatest at the head and/or toe, it is advisable to increase the amount of lateral reinforcement at the ends of the pile. Glanville et al.[20] recommends that the lateral reinforcement be not less than 1 per cent of the gross pile volume for a length of $2\frac{1}{2}$ to 3 times the diameter at each end and 0.4 per cent through the mid-body with a transition over a length of 2 to 4 feet. The Portland Cement Association recommendations are not as conservative as those listed above.[11, p.29] They recommend increased lateral reinforcement for a length of one diameter at the head and two diameters at the toe. If hollow piles are used, it is advisable to make the ends solid for a length of $2\frac{1}{2}$ to 3 diameters.

The tensile stresses during driving are not normally of concern in regard to failure, since practically all driving failures are the result of compressive stresses. However, in regard to durability tensile stresses high

enough to cause cracking of the concrete will provide entry of salt water to the reinforcing steel. During the early, easy driving it is advisable to use a soft cushion and/or a short stroke or short drop of the hammer. Except in conditions of very hard driving where the resistance is primarily located at the toe, the stresses at the head will be greater and failure is more likely to occur near the head. A heavy hammer with a smaller drop (or stroke) will give lower stresses than a light hammer with a high drop. A soft cushion will have less effect on reducing the efficiency of a heavy hammer than of a light hammer. Therefore, for maximum penetration with a given head stress, a heavy hammer with a relatively soft cushion should be used.[20] Since it is desirable to have a soft cushion during the initial stages of driving, a new cushion of two to four layers of soft wood (Douglas Fir or Southern Pine) should be used for each pile and discarded after one use. Such a cushion will get harder as the driving progresses and will transmit a greater proportion of energy as the pile reaches final penetration.

4.6 Precasting

In all concrete pier construction, serious consideration should be given to the use of precast members. The possible saving in time when precasting is used is obvious, but it does not mean that precasting will be

advantageous for all jobs. Ben C. Gerwick listed the following general principles which are necessary for a successful precasting operation:(19)

1. Central casting yard located with respect to transportation facilities.

2. Substantial base for casting beds and adequate storage space for aggregates and finished units.

3. Equipment for handling loads up to 25 tons must be available. Where units are too large to lift, there must be provision for launching and floating into place.

4. Experienced foremen and skilled crews are necessary.

5. Supervisory personnel must be well trained and must be capable of executing accurate construction and must maintain careful scheduling.

4.7 Prestressed Concrete

Much of the previous discussion on reinforced concrete would apply equally well to prestressed concrete. However, some engineers will maintain that prestressed concrete exhibits properties so radically different from ordinary reinforced concrete that it should be discussed as an entirely different product. Prestressed concrete does not have a sufficiently long service record to "prove" its durability but the prevalent feeling among engineers is that it should prove to be a far more durable material than either steel or conventional reinforced concrete.

In the first place, prestressed concrete requires a very high quality concrete--usually a 28-day strength in excess of 5,000 psi; and secondly, proper design can eliminate tensile cracks. Any cracks which might occur during handling or driving of piles will be closed by the prestressing force and there is a strong probability that the crack will actually bond together. By simple logic these facts combine to indicate the most durable product available to the engineer today.

Prestressed concrete allows greater use of precast elements than conventional reinforced concrete. The units can be larger and are more easily handled. Precast elements create difficulties in obtaining continuity, but many techniques have been developed to obtain either partial or complete continuity over supports.[29] The use of cantilevered beams with alternate suspended spans is one effective method. Prestressing for positive moment and using mild steel with cast-in-place concrete at the supports is effective in obtaining some degree of continuity. Post-tensioning successive beams placed end-to-end is a possibility but the anchorage of the prestressing tendon to the previously placed beam is difficult. For the deck slab, precast stringers placed side-by-side with a composite poured-in-place wearing surface, which contains mild steel reinforcement for negative moment, has been used successfully. This method is often accompanied by transverse

prestressing to tie the individual stringers together. The poured-in-place wearing surface provides a smoother surface for fork truck operation.

There is still need for improvement in the method of connecting piles to pile caps. Present practice is to use cast-in-place, posttensioned pile caps. It may be possible to use precast caps with bonding agents and post-tensioning or use embedded steel plates which are welded and then covered with grout for protection. Precasting of caps would require extreme accuracy in placing of piles and may in the long run be of no economic advantage.

It is probably in the manufacture of piles that prestressed concrete exhibits the greatest advantage over reinforced concrete. Handling stresses are not as critical and longer lengths with smaller cross-section can be used. Pretensioning seems best adapted to smaller size piles (up to 24 inches). The Raymond Concrete Pile Company has very successfully used posttensioning in 36 in. and 54 in. diameter hollow piles. The piles are cast in sections 16 feet long by a spinning process essentially the same as used in the manufacture of concrete pipe. To assemble a pile, the ends of the individual sections are coated with a resin bonding agent and then posttensioned together with eight to sixteen cables. These piles have been driven in lengths up to 208 feet.

4.8 Steam Curing

Steam curing is most frequently used as the means of obtaining high early strength of concrete to enable rapid reuse of stressing beds. The recommended cycle is three to six hours with no steam for initial set, then steam is applied for about 16 hours, maintaining a temperature ranging from 140° to 165°F, followed by a slow cooling period of three hours to prevent rapid shrinkage and drying.[26] This treatment will result in 50 to 70 per cent of the 28-day compressive strength. The bed should be so designed that steam is admitted uniformly throughout the length. The steam is confined by canvas or sheet metal hoods; insulated sheet metal is preferable for cold weather. There seems to be some difference of opinion on the use of calcium chloride as an accelerator for prestressed concrete work due to possible corrosive effects on the steel tendons. Since the value of calcium chloride as an accelerator reduces with an increase in temperature, there is no advantage where steam curing is used. Also, since the possibility of corrosion has not been ruled out, the conservative approach would be to not permit its use.

4.9 Recommendations for Durable Concrete

In summary, the following are recommended for durable concrete in a marine environment. These recommendations are basically as set forth by Wakeman et al.[52] but

with modifications deemed appropriate in the light of information presented by other authors.

1. Minimum cement content of 7 to $7\frac{1}{2}$ sacks per cubic yard. Minimum compressive strength of 4000 to 5000 psi for reinforced concrete, 5000 psi for prestressed concrete.

2. Maximum water content of 5 to $5\frac{1}{2}$ gallons per sack of cement. Slump should be less than 3 inches.

3. Use Type II cement with C_3A content below 8 per cent as specified in ASTM Designation C 150-56.

4. Use 3 to 6 per cent entrained air if structure is subjected to freezing and thawing action.

5. Use clean, non-reactive aggregates free from organic impurities, well graded with sufficient fines to give maximum density. Fines must be non-plastic.

6. Embed reinforcing steel with a minimum concrete cover of $2\frac{1}{2}$ to 3 inches for reinforced concrete and 2 to $2\frac{1}{2}$ inches for prestressed concrete.

7. Use clean water for mixing.

8. Thoroughly cure the concrete. After curing, air dry piling as long as practicable before driving.

9. Use particular care in handling piles to prevent damage.

4.10 Details

Piles are manufactured with square, round or octagonal cross-section. From the standpoint of durability the round shape is best with octagonal a good second choice. The flat slab design for decks has proved less subject to deterioration than the beam and girder type. The designer should always strive for simplicity and should avoid intricate form work.

5. STEEL FOR PIER CONSTRUCTION

5.1 Corrosion of Steel

The corrosion of steel (and almost all other corrosion processes) is electro-chemical in nature. Anodic and cathodic areas are established by different types of metal, impurities in a metal, variations in the surface such as welds or breaks in mill scale, stray currents, local differences in the concentration or temperature of the electrolyte, or by variations in the concentration of oxygen in the water.(14) The current flows from the anode through the electrolyte to the cathode and back through the metal to the anode to complete the circuit. The anode is an area of oxidation and the cathode is the area of reduction. At the anode the atoms of metal are converted into positively charged ions by the loss of electrons. The metal ions then pass into solution and form oxides and hydroxides which are normally deposited as rust near the anodic area. Some salts may also be formed which either stay in solution or are also deposited near the anode.

Hydrogen ions accumulate at the cathode and if they are not removed the corrosion process will stop. In an acid electrolyte the hydrogen is evolved as gas. Sea water is normally alkaline and the hydrogen ions are removed by reaction with oxygen to form water. The rate of corrosion in sea water is controlled by the amount of

oxygen available and increases with an increasing amount of oxygen.[23] This is not to imply that corrosion cannot occur in the absence of oxygen. Certain sea and harbor bottoms contain anaerobic, sulphate-reducing bacteria which decompose sulfates in the presence of iron and release oxygen which is then available to combine with the hydrogen ions. Corrosion by such bacteria is known as microbiological corrosion.

The pattern of corrosion on a steel pile can be extremely variable but, in general, corrosion below the mud line will be negligible; there will be an increasing rate of corrosion toward the surface of the water, reaching a maximum at about MLW. In most harbors there will be floating oil which will coat the piling and there may be relatively little corrosion in the tidal zone. The splash zone just above MHW will be the area of most active corrosion and the area above the splash zone will have a reduced rate of corrosion.

Highly alkaline water reduces corrosion and industrial acid waste may greatly accelerate the process. Structures located near the effluent of certain industrial plants may exhibit the most active corrosion near the mud line where the heavier acid wastes tend to settle. Such conditions can be dangerous because the corrosion is least suspected and least likely to be detected if it is below the water line.

Marine growth slightly retards corrosion by the exclusion of oxygen but, on the other hand, it will be destructive to paint or protective coatings. Some types of marine growth apparently exude acids which may cause severe pitting.

Laboratory tests and generalizations may be extremely misleading in attempting to estimate the useful life of a steel structure in a particular environment. The best source of information is the past history of structures in the particular area or from specimens exposed under natural conditions.

5.2 Cathodic Protection

Since practically all corrosion is electrochemical, it follows that if the entire structure is made cathodic with the current flowing to the exposed metal, no corrosion can take place. Or, stating it another way, the current is prevented from flowing away from the metal surface. Prevention of corrosion by such an impressed current is cathodic protection and is accomplished by utilizing galvanic anodes of a material which is more negative on the electromotive force series than the metal being protected and which provides its own source of energy, or by providing a voltage from an external source such that current flows into the structure.

Galvanic anodes are normally of a magnesium-aluminum-zinc alloy. The anodes are placed on the harbor bottom or suspended in the water beneath the pier and electrically connected to the steel surfaces to be protected. The galvanic anode method of protection is more advantageous for small structures since installation is simple, no external source of power is needed and relatively little maintenance is necessary, except for the replacement of anodes as they are consumed.

For large structures the impressed current method will probably prove the most economical. The impressed current system consists of a ground bed, or system of anodes, and a source of direct current. The ground bed can be made of scrap iron or steel which corrodes and must be replaced periodically or any of a number of materials such as graphite, high silicon cast iron or platinum-palladium alloy, which are not readily corroded.[21] A direct current source may be utilized if available, but more commonly the current will be provided by rectifiers and alternating current source.

When a cathodic protection system is first placed in service, the alkalinity at the metal surface will cause a layer of calcium carbonate and magnesium hydroxide to be formed. This layer is relatively insoluble and will provide additional protection to the steel and, after the layer is formed, the current requirement will be less. In

general, piled structures require 5 to 12 milli-amperes per square foot of exposed steel for the first few months, and 3 to 6 milli-amperes per square foot for permanent protection. Current requirements can vary over wide ranges as listed below:[22]

Bare steel in sluggish water containing only a small amount of oxygen - - - - - - - - -	- 1-5 Ma/ft^2
Bare steel in highly aerated, rapidly moving water - - -	- 20-100 Ma/ft^2
Bare steel in soil - - - -	- 0.1-5.0 Ma/ft^2

The resistivity of sea water is normally 20-30 ohm-cm, thus requiring an operating voltage of 5 to 25 volts.

Even though a great deal is known about the corrosion process, actual design of a cathodic protection system is largely empirical. No attempt should be made to design a cathodic protection system without the services of an experienced engineer who specializes in that field and who is familiar with the local area.

It must be kept in mind that cathodic protection is effective only for the submerged portion of the pile. This may mean that the area of greatest corrosion (splash zone) receives no protection. Protection above the mid-tide level must be provided by paints, coatings or concrete jackets. In fact, it is desirable to have the entire pile initially protected by a coating even if cathodic protection

is used. A coating will cut down on the initial current requirements and the cathodic system will provide increasing protection as the paint or coating deteriorates.

5.3 Paint, Tar and Bituminous Coatings

Chlorinated rubber and phenol-formaldehyde resin type paints are giving good service and there are many proprietary paints of this type available. Tars and bitumens also give good results. Coating of the submerged part of piling is strictly a one-time application as it can never be renewed nor can it be patched after the pile is driven. A certain amount of damage to an applied coating is unavoidable in handling and driving of the piles. As to whether coating or painting of piles is economically justified, only local experience can be used as a guide. All that can be said is that it will somewhat prolong the life of the piling. For steel portions of a pier other than the piles, the answer is simple--all exposed steel must be protected by painting.

5.4 Concrete Jackets

Concrete jackets are very effective in protecting steel piles from corrosion. The methods of jacketing piles are basically the same as those used for protection of wood piling discussed in chapter 3, except that gunite and cast-in-place jackets appear to have been used more frequently than precast cylinders. The jackets may extend all the way

to the mud line but often they extend only from the mid-tide elevation up through the splash zone.

Piles located in the surf zone may suffer severe corrosion at the ground line. This is due to the abrasive action of the sand which removes the rust as fast as it forms and continually exposes bare metal to further attack. It has been found that cylindrical piles are less subject to this abrasive action than H-piles.[45] Piles located in the surf zone should be protected by concrete jackets or steel cylinders extending a few feet above and below the sand surface. Creosoted wood jackets held in place with galvanized steel bands are also effective.

5.5 Details

There is little to choose between the different steel pile shapes in normal pier construction. H-piles can penetrate hard strata better than other shapes. Pipe piles can be filled with concrete for additional strength and less loss of strength if the shell corrodes. Other shapes have been used, including I-beams, piles assembled from old rails and box piles assembled by welding sheet piles together.

Bracing should be eliminated if possible. It is particularly susceptible to corrosion since relatively thin members are used. The crevices and cuts at the joints are difficult to protect and will cause increased maintenance

costs for painting. If bracing must be used to reduce the unsupported length of piles, heavy members should be used, it must be well protected by painting and it must be kept above the splash zone.

The loss of strength due to corrosion should not be passed over lightly. A small loss in cross-section (as in the flanges of an H-pile) can result in a very large loss in the supporting capacity due to the reduced moment of inertia.

It would appear that the ultimate protection for steel piling is to use cathodic protection and concrete jackets from the mid-tide elevation to the top of the pile.

6. DESIGN LOADINGS

6.1 Vertical Loads

A pier must be designed to withstand both vertical and horizontal loads. Vertical loadings do not present a very difficult problem. Normally design loads can be determined accurately or can be quite accurately assumed on the basis of past experience. Such loadings include dead weight of the structure, weight of cargo stored on the pier, wind and snow loads, crane loads and truck and train loads. Dead weight of the structure is subject to exact determination. Cargo loadings are determined on the basis of the type of cargo to be handled. For general cargo this is usually assumed as 500 to 600 pounds per square foot. Crane loads must be determined for the individual case based on the type of crane and the loads which will be lifted. Under present-day operations, the truck loading for a general cargo pier is assumed as H20-S16 and train loading is Cooper E-50. Since speeds will be low on the pier impact of 10 to 20 per cent is sufficient. Fork trucks cause concentrated loads since most of the weight is carried by the front wheels, but normally for a general cargo pier either the uniform load or the H20-S16 will be critical. Wind and snow loads must be based on local conditions and are usually covered by code.

Deflections due to vertical loads are due to compression in the piles and elastic deformation of the soil, and are of such small magnitude that they are ignored for practical purposes. This assumes, of course, that neither the pile nor the soil is overloaded.

6.2 Horizontal Loads

Horizontal loads are not subject to an accurate analysis nor can the deflections due to horizontal loads be ignored. The sources of horizontal loading are impact of berthing ships, wind, current, floating debris or ice, wave pressure and earthquake. Most of the information on horizontal loading is of an empirical nature, both as regards the loading itself and the ability of the structure to resist the horizontal load. In any case, a correct analysis must consider both the structure itself and the soil into which the piles are driven, although one can find analyses where the soil is entirely ignored.

6.3 Ship Impact

There are many variations in the analysis of the berthing impact of ships but basically the study involves equating the kinetic energy of the moving ship to the work done through a finite deflection of the pier and the fender system. The ship normally approaches the pier at some angle (usually between 20° and 30°), strikes the pier at a point of contact fairly far forward on the ship, then

shears off and comes to rest approximately parallel with the pier. Some time in this interval, mooring lines will have been passed to the pier and will have been snubbed to help arrest the forward motion of the ship. Only in the case of accident will the ship strike the pier head-on, a condition for which the pier and fender system cannot be economically designed, and an approach parallel to the pier with no impact would be rare indeed.

The energy of the moving ship is absorbed by:

(1) the deformation or deflection of the pier and the fender system;

(2) friction of the ship on the fender system;

(3) the pull on the mooring lines;

(4) frictional resistance of the underwater portion of the hull;

(5) backing power of the ship's screws, or tugs;

(6) imparting a rotational or yawing velocity to the ship due to the fact that the point of impact will not coincide with the center of gravity of the ship;

(7) elastic and/or plastic deformation of the ship's structure.

It is obvious that not all the energy of the ship must be absorbed by the pier itself. There is no exact mathematical solution which can account for all the variables, so it is necessary to rely on empirical rules.

The mass of the ship will be known or can be quite accurately estimated from the knowledge of the type of ship which will be using the pier. The velocity of the approaching ship and the relative amount of the ship's energy actually imparted to the pier can only be roughly estimated. Minikin[32, p.188] gives the results of 49 observations of approach velocities in a variety of ports. With only one exception, the velocity component normal to the pier was less than one foot per second and in 88 per cent of the cases it was less than one-half foot per second. Minikin also noted that the approach velocity of large vessels is normally lower than that of small vessels. This, coupled with the fact that large vessels will normally be aided by tugs and small vessels may not, leads to the logical assumption that within the normal range of vessels berthing at a particular pier, the total energy will not vary greatly with variation in the size of ship. Therefore, the design should be based on the ship which is most frequently berthed.

Minikin states the amount of the ship's energy which actually is transmitted transversely to the pier cannot exceed one-half of the transverse component of the kinetic energy of the ship.[32, p.168] He reasons this from the fact that the point of contact can be no further aft than the point where the straight side of the ship starts to curve into the bow. The point of contact will

then be approximately one-half the distance from the center of gravity to the bow and the amount of energy transmitted will be in proportion to this distance, i.e. one-half of the transverse energy. This line of reasoning may be subject to criticism, but his results do agree with observed conditions and with experimental results.

Baker recommends that four-tenths of the transverse energy be used in the design of fendering.(6) This recommendation was based on observations of a bell dolphin at an oil pier in England where 15,000 ton tankers were berthed in rather severe conditions of wind and tide. Levinton in American Civil Engineering Practice(1, p.21-63) recommends that the entire transverse kinetic energy be used in design. In view of the information submitted by other authors, this appears over-conservative. Cornick recommends 40 per cent of the transverse energy for lateral blows and only for the case of head-on collision should the full kinetic energy be assumed.(15)

When a vessel contacts the pier, the resistive force will increase from zero to a maximum when the velocity of the ship reaches zero. Therefore the work done will be one half the maximum force multiplied by the distance moved, i.e.

$$\frac{Pd}{2}$$

Where "P" is the maximum force and "d" is the total deflection of the structure. Equating this to the effective

kinetic energy of the ship, the equation used for design will be:

$$\frac{Pd}{2} = k \frac{1}{2} \frac{w}{g} v^2 \quad \text{or} \quad Pd = k \frac{w}{g} v^2$$

Where "k" is a factor for the portion of the transverse kinetic energy which is effective in deflecting the structure, "w" is the gross weight of the ship and "v" is the component of the ship's velocity normal to the pier. For normal conditions, it is recommended that the following be used:

$$k = 0.40 \quad ; \quad v = 1.0 \text{ ft/sec.}$$

If it is known that berthing will always be done in calm conditions, v = 0.5 ft/sec. might be used.

It is obvious that in the event of accidental head-on collision the pier should not collapse. Local damage to both pier and ship is to be expected. There is no information available as to what factors to use, but k = 1.0 and v = 3.0 to 5.0 ft/sec. appear reasonable. For such an analysis, yield point stresses should be used.

6.4 Wind Forces

The lateral forces due to wind are usually covered by local codes and depend on the maximum wind velocities in the area. In the absence of such codes, the wind force must be estimated by the use of empirical formulas using known wind velocities. Chellis recommends the formula:

$$p_W = .004\ Vw^2 \qquad (9, p.171)$$

Where p_W is the horizontal wind load in pounds per square foot, and Vw is the wind velocity in miles per hour. The wind pressure is applied to the vertical projection of the pier and pier shed plus the exposed presentment of the ship which is not shielded by the structure.

Cornick(15, p.203) presents the following formula which was proposed in a paper presented at the 18th International Congress of Navigation:

$$F = K\ S\ (V - U)^2$$

Where: F is the total force on the vessel in kilograms;

S is the vertical projected surface of the ship in square meters;

V is the velocity of the wind in meters per second;

U is the velocity of the ship in meters per second;

K is a constant which has values between .07 and .08.

Since the drifting speed would be zero when the ship was moored, the expression reduces to:

$$F = K\ S\ V^2$$

which is very similar to the one recommended by Chellis.

Levinton presents the following table of wind loads in American Civil Engineering Practice.(1, p.21-61)

Vw(mph)	Pw(lb./sq.ft.)	Vw(mph)	Pw(lb./sq.ft.)
10	0.5	65	21.6
15	1.2	70	25.2
20	2.0	75	28.8
25	3.2	80	32.7
30	4.6	85	36.9
35	6.3	90	41.4
40	8.2	95	46.2
45	10.4	100	51.1
50	12.8	105	56.4
55	15.5	110	61.9
60	18.4		

The above figures were calculated on the basis of an air density of 0.07651 lb./cu.ft. at 15°C, 760 mm Hg and assumed shape factor of two for the long nearly prismatic body of the ship or wharf. Wind velocities are normally measured high above the ground so, to allow for the reduced velocity on a low structure, the above values can be reduced by a factor of 0.8.

Some authors consider the actual static force of the wind acting on the ship as being negligible in comparison with the dynamic impact against the pier due to the velocity imparted to the vessel by the wind.(51) While this is undoubtedly true, it hardly seems worthwhile to evaluate this force in view of the empirical nature of the impact calculations. It must also be remembered that a ship will not be maneuvered into a berth during a period of high wind.

The above empirical methods do not take into account the effect of the shape of the vessel or of the various projections above the deck. Following World War II the U. S. Navy conducted model tests on single vessels and groups of vessels to determine the effects of wind and current on moored ships.[5] These tests, the results of which are presented in NavDocks TP-Pw-2,[33] revealed that the wind produced not only lateral and longitudinal forces but a yawing moment as well. The tests also revealed that the shielding effect of ships moored side-by-side was considerable. The total force on six ships was only 50 per cent greater than the force on a single ship. This is of importance in evaluating the forces on a ship which may be shielded by the pier and transit shed. Naval vessels of the submarine, destroyer, cargo, aircraft carrier and floating drydock types were tested. Reference 33 gives charts and tables which can be used to extrapolate the test results. Such extrapolation is limited to ships with superstructures similar to the model tested.

For design with wind loads combined with normal loading, the stresses can be increased in accordance with the code which applies to the material being used.

6.5 Current Forces

Information on the forces of a moored ship due to currents is very limited. This is probably explained by

the fact that most facilities are constructed in areas where there is little current, hence the forces due to current are of little interest to designers. Chellis(9, p.172) gives an empirical formula which is based on the assumption that the unit pressure due to the current is a function of the equivalent static head corresponding to the current velocity, i.e. $H = Vc^2/g$. The total force as given by Chellis is as follows:

$$Fc = k\ B\ Vc^2$$

Where: H is the head in feet;

Vc is the velocity of the current in feet per second;

g is the acceleration of gravity in feet per second^2;

Fc is the total force in pounds;

B is the projected area of the structure or the hull below the water line in square feet;

k is a form factor: 1.0 for round piles, 1.4 for square piles and bracing, 0.82 for current normal to the centerline of vessel and 0.15 to 0.60 (depends on lines of vessel) for current parallel to the vessel.

Mean velocity usually occurs at about 0.6 of the depth, surface current is normally 0.85 of the mean velocity, and the maximum velocity is about 1.1 times the mean velocity.[9]

The maximum velocity occurs somewhere between the surface and one-half the depth.

Levinton(1, p.21-63) gives basically the same formula but the breakdown of the coefficient "k" is not as detailed as that given by Chellis.

Chellis(9, p.172) also gives formulas for friction drag and propeller drag when the current flows parallel to the vessel. For friction drag:

$$F_d = .007\ S\ Vc^2$$

Where: F_d is the total force in pounds;

S is the wetted area of the hull in square feet;

Vc is the velocity of the current in knots (1 knot = 6080.2 ft/hr).

Levinton(1, p.21-63) gives the same formula with a coefficient approximately equal to 0.01.

For propeller drag, Chellis gives:

$$P_d = 2.88\ Ap\ Vc^2$$

Where: P_d is propeller drag in pounds;

Ap is the projected area of the propellers in square feet;

Vc is the velocity of the current in knots.

The tests reported by Ayers and Stokes(5) indicated that, as in the case of wind, the forces due to

current consisted of lateral and longitudinal forces and a yawing moment. The tests verified that the force was approximately proportional to the square of the velocity, but the forces also varied approximately in inverse ratio to the depth of the water. None of the empirical formulas take the effect of the water depth into account. As in the case of wind, there is considerable shielding when a group of vessels are moored side by side. The resistance of two ships was only about 20 per cent greater than a single ship and for six ships about 120 per cent greater.

It must be realized that the previously listed formulas for forces due to wind and current are empirical and give only approximate results at best. They are adequate for the general design situation where the principal forces are other than those caused by wind and current. However, where it is necessary to estimate the current and wind forces with some degree of accuracy, model tests are the only practical solution.

6.6 Wave Forces on Piling

Wave forces on piling have been evaluated and laboratory tests have been made to evaluate the necessary coefficients. The procedure presented by Morison(34) was developed on the assumption that the wave form is trochoidal and the partial velocities and accelerations are sinusoidal. These conditions are approximately true for most deep water

[illegible]

[illegible]

[illegible]

[illegible]

waves or shallow water waves of small steepness. The procedure is not applicable to piling located in the zone of wave breaking. The design procedure is not complicated, but the formulas are rather involved and certain tables are necessary for the calculation so the reader is referred to either reference 8 or 34, both of which present Morison's procedure. Reference 35 presents some more recent test results and makes minor revisions to the design procedure.

Wave forces on a moored ship and forces on a ship due to surging are complete studies, and will not be discussed in this thesis.

6.7 Earthquakes

Earthquakes are one further source of lateral forces. Earthquakes are confined to certain regions and generally local codes will give design requirements. Normally some percentage (approximately 10%) of the vertical dead and live load is assumed as the horizontal force. The piles are checked for shear, moment and deflection, considering the force as a static force applied at the center of gravity of the structure. If the deflection thus calculated is large, some designers assume a maximum deflection (say two inches) based on the observed maximum horizontal amplitude of earthquakes in the region. The reasoning here is that if resonance is avoided, the deflection of the pier cannot exceed the horizontal amplitude of the ground quake.

Since a pile-supported pier will be flexible, it will have a natural period of vibration which varies with its mass, the unsupported length of the piles and the conditions of fixity at the head of the piles. It is imperative that the structure does not have a natural period of vibration which falls within the normal range of the periods of earthquakes.

Chellis(9, p.163) gives the following formula for the period of vibration of a single pile:

$$T = c_f \quad \frac{W\ lu^3}{E\ I}$$

Where: T is the period of the vibration in seconds;

W is the vertical dead and live load on the pile in pounds;

lu is the unsupported length of the pile in inches;

c_f is a constant dependent on end conditions and is approximately 0.18 for free end at top of pile
0.09 for full fixity at top of pile
(actual conditions will always be between these values).

For a structure supported on piles of varying length, the formula becomes:

$$T = c_f \quad \frac{\sum W}{\sum \frac{E\ I}{lu^3}}$$

The above formulas would not apply to a pier with batter piles, but batter piles increase the natural frequency so much that resonance is not of concern.

Chellis gives the range of commonly observed periods of earthquakes as 0.1 to 0.7 second at locations near the epicenter, with the largest acceleration occurring in waves having periods less than 0.5 second. Periods as high as 2 or 2.5 seconds are possible but the destructiveness of the quake is approximately inversely proportional to the period so the higher periods are not as dangerous. It appears reasonable that if the structure had a natural period less than 0.1 second or greater than 2.5 seconds, resonance is not likely to occur.

6.8 Fixity of Pile Ends

One of the most perplexing problems in evaluating lateral forces or in determining the unsupported length of piles is the determination of the degree of fixity at the ends of piles. Two feet of embedment of wood or steel piles in concrete is considered sufficient to give full fixity. If a concrete pile is built-in at the top with properly embedded bars bent into the slab or beams, it can be considered completely fixed. Piles braced in both directions, with lower ends of bracing fastened more than five feet from the top of the pile, are considered fixed at the bottom of the bracing, although this is not strictly

correct. Where there is a question of the proper degree of fixity, calculate stresses for both free and fixed conditions and design for the most unfavorable.

Normal piles can be considered fixed between the dredge line and five feet below the dredge line in firm sand or gravel. For soft material or loose sands and silts, the point of fixity will be between five and ten feet below the dredge line. For very stiff piles, such as the Raymond cylinder piles, the above figures may be doubled. It would be best to conduct a lateral load test and make an analysis as discussed in section 2.7, only then could realistic values be determined.

If batter piles are used for lateral loads, it is common practice to ignore the lateral capacity of the vertical piles. The necessary deflection to mobilize the full capacity of batter piles is much smaller than that required to mobilize the lateral capacity of a vertical pile. Consequently, the batter piles actually carry the major part of the load and the additional capacity of the vertical piles is simply an additional factor of safety.

6.9 Fenders

A subject which has not been discussed in this thesis but is, none the less, a very important consideration in the design of piers is the fender system. The fenders must be designed in conjunction with the pier,

since the lateral load from ship impact will depend on the total amount of deflection of the fender system and the pier structure itself. Fenders vary from practically unyielding rubbing strips attached to the side of the pier to very elaborate devices using dash-pots and springs for energy absorption. The choice of fender will be governed by the conditions of tide, wind and current at the pier site. Let it suffice to say the system chosen should be as simple as possible consistent with the energy absorption required at the particular site; elaborate systems requiring lubrication and/or frequent maintenance should be avoided.

7. COST FACTORS FOR PIER CONSTRUCTION

A discussion of costs necessarily involves so many variables that it is nearly impossible to be specific. In fact, if an attempt is made to discuss specific costs, one would be restricted to a particular pier at a particular site. The following paragraphs attempt only to give a general discussion of the factors involved in estimating the cost of a pier.

One of the most important considerations and also one of the most difficult to assess is the workload of the contractors in the local area. During slack seasons bids will be lower and contractors may take jobs at cost simply to keep a crew mobilized. The general fluctuations of the entire economy has an indirect influence in this regard, since industrial plant expansion will be motivated by periods of good business. Since there may be from one to three years involved in the planning and design of facilities, the fluctuations of the construction industry will somewhat parallel but will lag behind the fluctuations in the general economy. Private projects usually take 18 months or less for planning and design and public works, since financing is more cumbersome, usually take two to six years.

Availability of equipment is an extremely important influence on the cost of construction. A contractor

who is already mobilized for a particular type of work can underbid another contractor who must mobilize from "scratch." On the other hand, for some very large, unique jobs it is entirely feasible, sometimes mandatory, to build a specialized piece of equipment to be completely amortized on that one job. One reason that Raymond's prestressed cylinder piles are not used more widely is that one very large job is necessary to justify the initial cost of setting up the plant and mobilizing the equipment; and thereafter they can be economically used on smaller jobs. The same applies to prestressed piles of any size. If the job is big enough, prestressing beds can be built specifically for that job, but prestressed piles might be uneconomical on a small job unless a commercial prestressing plant is close enough to the job site to make transport of the finished piles feasible.

Mobilization costs are dependent on the distance the equipment must be moved, whether the equipment is wheeled or tracked, freight rates if the equipment is moved by rail, and towing costs for floating equipment. For one particular job and one particular contractor, the mobilization cost will be a constant, which must be distributed over the entire job. Quite obviously, then, the larger the job, the less important will be the mobilization cost.

In certain locations it may be necessary for the designer to select a type of pile or type of deck which in

itself is more expensive but which, in the end, will result in a less costly job if it fits the equipment and the skills of the local contractors. This is not to say that a designer should hesitate to introduce a new product or new technique simply because it has not been used in that area; contractors are an extremely versatile group.

Labor strikes are still another variable and again one impossible to assess accurately. Availability of materials is of utmost importance and steel strikes, rail strikes, etc. will influence the designer in choosing the material to be used. Unfortunately, the timing of such strikes is usually such that the designer can do little more than "hope for the best."

On a particular job many variables are involved. The length of piles has a very definite influence. The actual labor cost of driving a pile is more or less independent of the length for normal jobs, but wood piles are proportionately more expensive in lengths over 50 feet. Usually steel piles over about 60 feet in length must be spliced. The cost of splicing can vary over wide ranges, depending on whether it is a simple butt weld or requires splice plates. The cost of concrete piles also increases rapidly in lengths over 60 feet. The actual labor involved for driving steel or concrete piles will be about double that for wood piles due to the heavier weights involved. If piles are widely spaced the cost per pile will be

somewhat higher due to the additional maneuvering involved.

From the preceding paragraphs it is evident that generalizations regarding cost of construction are not possible, but it is also obvious that the designer must have a thorough knowledge of the contractors in his area as well as an understanding of the general fluctuations in the construction market. The practical solution for obtaining the lowest cost construction is to provide for alternate bids. The designer can eliminate those types of construction which are obviously uneconomical and then the final determination will be made on the basis of the contractors' bids.

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